International Conferences on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

2001 - Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics

29 Mar 2001, 4:00 pm - 6:00 pm

Seismic Active Earth Pressure Considering Effect of Strain Localization

Jian-Min Zhang
Tsinghua University, Beijing, China

Deji Li
Tsinghua University, Beijing, China

Proceedings: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics and Symposium in Honor of Professor W.D. Liam Finn
San Diego, California, March 26-31, 2001

Seismic Active Earth Pressure Considering Effect of Strain Localization

Jian-Min Zhang
School of Civil and Hydraulic Engineering
Tsinghua University
Beijing 100084, China

Deji Li
School of Civil and Hydraulic Engineering
Tsinghua University
Beijing 100084, China

ABSTRACT

A new method, which is based on pseudo-static and limit-equilibrium analysis, was proposed for evaluating seismic (static plus dynamic) active earth pressures induced by backfill soils behind movable rigid retaining walls. It has the advantage over the Mononobe-Okabe method since it can take into account the effects of strain localization and post-peak reduction in the shear resistance that occur in the denser backfill soil during a strong earthquake.

INTRODUCTION

Several approaches have been developed to determine earth pressures against gravity retaining walls during earthquakes. Among these, the Mononobe-Okabe method (1924) based on pseudo-static and limit equilibrium approach has been widely used. However, this method cannot consider the effects of strain localization and reduction in the shear resistance from peak to residual state that occur in the denser backfill soil during a strong earthquake. Koseki et al (1998) developed a graphic procedure to consider the effects of strain localization and reduction of post-peak shear resistance in the determination of the seismic active earth pressure.

This paper is to present a new method with a high degree of mathematical elegance for the evaluation of the seismic active earth pressure under considering the effects of strain localization and post-peak reduction in the shear resistance. The effectiveness of the proposed method was checked preliminarily with previous model test results.

EFFECT OF STRAIN LOCALIZATION

The effects of strain localization are hereafter referred to as formation of a shear band in the backfill soil behind a retaining wall and reduction of post-peak shear soil resistance in the shear band. The appearance of the shear band in the backfill soil may eventually lead to the formation of a sliding plane behind the retaining wall. If the sliding plane is induced by the displacement of the retaining wall away from the backfill soil, it is usually called "active failure plane". The active failure plane and the back surface of the wall create a triangular active soil wedge, as shown in Fig. 1(a) where an active soil wedges *ABC* is bounded by the back surface *AB* and an active failure plane *AC* passing through the toe of the wall. An active soil wedge can be formed, only when the wall displacement away from the soil becomes large enough to fully mobilize the shear resistance of the soil.

It is well known that the magnitude of active earth thrust against a retaining wall is related to the size and effective weight of the active soil wedge. The latter depends mainly on the shear resistances mobilized at the active failure plane and the back surface of the wall. For denser backfill soil, its shear resistance mobilized varies, from peak to residual value, with the increasing shear strain and the appearance of the shear band. Peak shear resistance is always first mobilized at a relatively small shear strain and residual shear resistance is then reached. It has been experimentally confirmed that a small displacement of about 10 times the mean diameter of sand particles in the direction parallel to the shear band is enough to reduce the mobilized shear resistance from peak to residual value (Koseki et al, 1998). Bolton and Steedman (1985) conducted dynamic centrifuge tests on retaining wall models and found that the mobilized shear resistance angle on a failure plane that is formed in the backfill sand reduces from 50 degrees to 33 degrees as long as a relative displacement of the order of 10 times the mean diameter of sand particles is triggered. This implies that a rational method of evaluating the seismic earth pressure should properly consider post-peak reduction in the shear resistance and thus the effects of strain localization occurring in the backfill soil.

In the determination of the seismic earth pressure, therefore, a rational value of the shear resistance angle of the backfill soil should be adopted. Figs. 1(a) and 1(b) show a significant difference in the active soil wedges corresponding to two different shear resistance angles respectively: peak shear resistance angle ϕ_p and residual shear resistance angle ϕ_r. In the Mononobe-Okabe method, the shear resistance angle is assumed to be constant within the whole backfill soil. The seismic earth pressure may be determined by the use of this method adopting ϕ_p for the case shown in Fig. 1(a) and adopting ϕ_r for the other case in Fig. 1(b). Based on the evaluation using the Mononobe-Okabe method, obviously, the earth pressure is underestimated for the former case and overestimated for the latter case. The real value of the seismic earth pressure may fall between those determined for the two cases. This is because the effects of strain localization are neglected in the determination of the seismic earth pressure.

In this study, the effects of strain localization and reduction of post-peak shear resistance are properly evaluated through considering the residual shear resistance mobilized on a previously formed active failure plane in the denser backfill soil. For the conditions where the other factors such as seismic coefficient are the same, it is assumed that the position of active failure plane is determined by the peak shear resistance, whereas the magnitude of the active earth pressure depends on the residual shear resistance mobilized on the same failure plane. Based on such explanation, a way to solve the seismic active earth pressure problem under considering the effects of strain localization is proposed as follows.

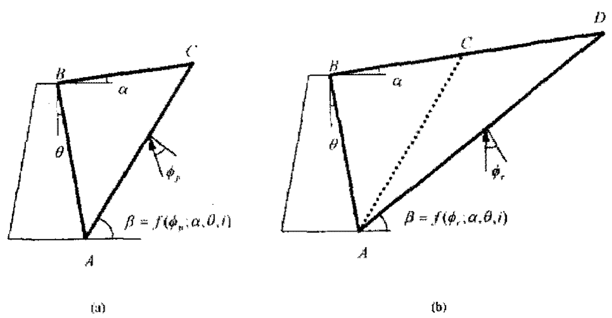


Fig.1. Active failure planes determined by peak and residual shear resistance angles respectively.

FORMULA OF SEISMIC ACTIVE EARTH PRESSURE

An active soil wedge of a static weight W with two inertial force components k_vW and k_hW in both vertical and horizontal directions, as shown in Fig. 2, was considered. Herein, k_v and k_h are, respectively, coefficients of vertical and horizontal seismic acceleration in a fraction of gravitational acceleration, usually called "vertical seismic coefficient" and "horizontal seismic coefficient". The resultant body force of the soil wedge W' does not always remain vertical during an earthquake. The angle between W' and W is called "angle of seismic coefficient" and defined as $i = \tan^{-1}$ $[k_v/(1-k_h)]$. k_v and k_h take positive signs in the upward and toward-wall directions. In addition, there exist two other external forces acting on the soil wedge: the total seismic active earth pressure P and the resultant reaction force F on AC.

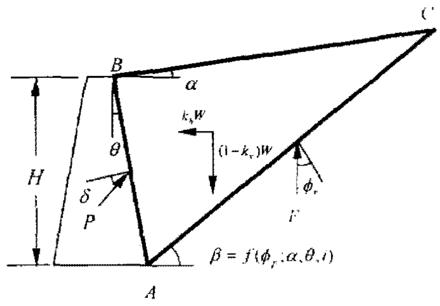


Fig. 2. External forces acted on active soil wedge

It should be pointed out that for the conditions where the other factors determining the active earth pressure during an earthquake are the same, the slope of active failure plane AC with respect to the horizontal, β, can be determined by the peak shear resistance angle ϕ_p, while the angle between the reaction force F on AC and the normal line of AC is assumed to be equal to the residual shear resistance angle ϕ_r. Based on such concepts as well as pseudo-static and limit-equilibrium approach, β and P were obtained by analyzing polygon of all the external forces acting on the soil wedge as shown in Fig. 3 and then by maximizing P with respect to β. Since the volume of this paper is limited, the specific derivation will not be described here.

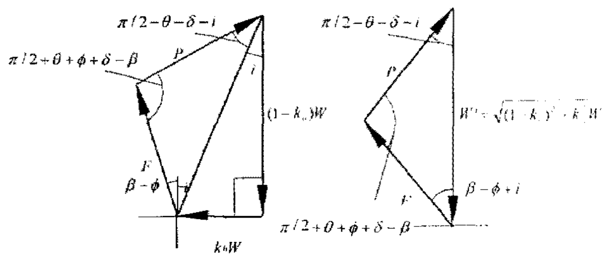


Fig. 3 Polygon of the forces acting on active soil wedge

As a result, the resultant seismic active earth pressure under considering the effects of strain localization, P, may be determined by the following formulas:

$$P = \frac{1}{2}\gamma(1-k_v)H^2K_a \tag{1}$$

$$K_a = \frac{\cos(\alpha-\theta)}{\cos i\cos^2\theta}\cdot\frac{K_1}{K_2} \tag{2}$$

$$K_1 = -(K\cos\omega_1 - 1)\cos(\phi_p - \phi_r) + K\sin\omega_1 \sin(\phi_p - \phi_r) + (K\sin\omega_2 - \sin\omega_3)\sin(\theta - \phi_r + i) \tag{3}$$

$$K_2 = -(K\cos\omega_1 - 1)\cos(\phi_p - \phi_r - \alpha - \delta - i) + K\sin\omega_1 \sin(\phi_p - \phi_r - \alpha - \delta - i) + (K\sin\omega_2 - \sin\omega_3)\sin(\theta + \phi_r + \delta - \alpha) \tag{4}$$

$$K = \frac{\cos^2(\phi_p - \theta - i)}{\cos(\theta + \delta + i)\cos(\alpha - \theta)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi_p)\sin(\phi_p - \alpha - i)}{\cos(\theta + \delta + i)\cos(\alpha - \theta)}}\right]^2} \tag{5}$$

in which $\omega_1 = \alpha + \delta + i$; $\omega_2 = \theta + \delta + \phi_p - \alpha$; $\omega_3 = \theta - \phi_p + i$; α, slope of the ground surface with respect to the horizontal; δ, wall friction angle; θ, slope of the back of the wall with respect to the vertical; and the sign convention is depicted in Fig. 2 where α , δ and θ are shown as positive.

It should be noted that:

1) $(\phi_p - \alpha - i) \geq 0$ and $(\theta + \delta + i) < 90^o$;

2) The peak shear resistance angle ϕ_p and the residual shear resistance angle ϕ_r are required to be determined using a shear test in plane strain. A simple method (Zhang et al, 1998a), by which ϕ_p and ϕ_r can be evaluated based on a conventional triaxial test, may be adopted;

3) i, k_r and k_h may be determined based on an equivalent seismic coefficient (Zhang et al, 1998b) for taking into account the non-uniform seismic acceleration distribution with height of the backfill soil;

4) Provided that $\phi = \phi_p = \phi_r$, the proposed formula (2) can be reduced to Eq. (6):

$$K_a = \frac{\cos^2(\phi - \theta - i)}{\cos i \cdot \cos^2\theta \cdot \cos(\delta + \theta + i)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot\sin(\phi - \alpha - i)}{\cos(\delta + \theta + i)\cdot\cos(\theta - \alpha)}}\right]^2} \tag{6}$$

Eq. (6) is the Mononobe-Okabe's dynamic active earth pressure equation.

In addition, the procedure of evaluating the seismic active earth pressure using the proposed formulas is better than the graphic procedure proposed by Koseki et al (1998), because it allows solutions to be obtained in a very simple, logical and mechanical procedure, and thus it is easy to be used.

Figures 4, 5 and 6 provide charts showing the relationships between the peak shear resistance angle ϕ_p and the seismic active earth pressure coefficient K_a with respect to specified values of the horizontal seismic coefficient k_h and the residual shear resistance angle ϕ_r, which were calculated by Eq. (2) for the conditions that $\alpha = \theta = \delta = k_v = 0$.

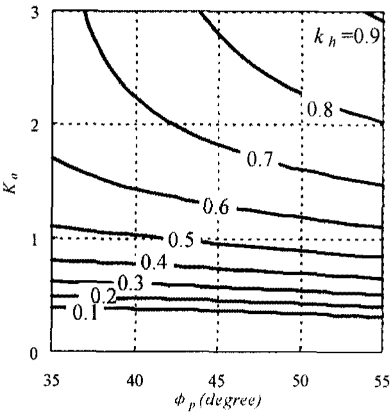


Fig. 4 Coefficient of dynamic active earth pressure for the conditions where $\phi_r = 28^o$ and $\alpha = \theta = \delta = k_v = 0$

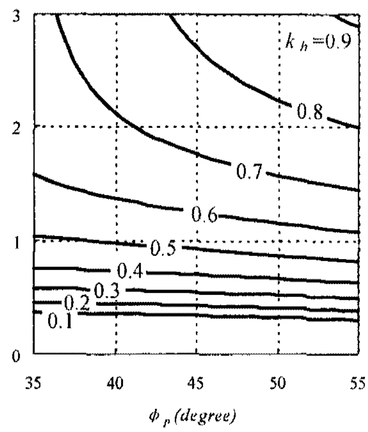


Fig. 5 Coefficient of dynamic active earth pressure for the conditions where $\phi_r = 30^o$ and $\alpha = \theta = \delta = k_v = 0$

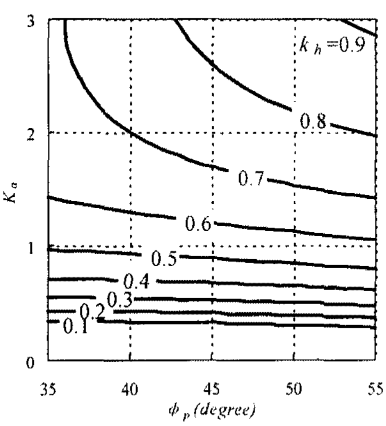


Fig. 6 Coefficient of dynamic active earth pressure for the

conditions where $\phi_r = 32^o$ *and* $\alpha = \theta = \delta = k_v = 0$

EFFECTIVENESS OF METHOD OF EVALUATION

The seismic active earth pressure may be determined using the method based on Eq. (1) through Eq. (5). This method has the advantage over the Mononobe-Okabe method since it can consider the effects of strain localization and post-peak reduction in the shear resistance that occur in the denser backfill soil during a strong earthquake. Presented in Fig. 7 is a comparison made between the relations of K_a with k_h, which were calculated respectively by the present method and the Mononobe-Okabe method using ϕ_p and ϕ_r. It is shown that for a specified value of k_h, the value of K_a determined by the present method is between the two values of K_a determined by the Mononobe-Okabe method for the cases where $\phi = \phi_p = 50^o$ and $\phi = \phi_r = 30^o$. In particular, the k_h - K_a relation curve determined by the present method becomes consistent with one of the other two relation curves determined by the Mononobe-Okabe method, when $\phi = \phi_p = \phi_r = 30^o$ or $\phi = \phi_p = \phi_r = 50^o$. This indicates the essential correctness of the proposed method.

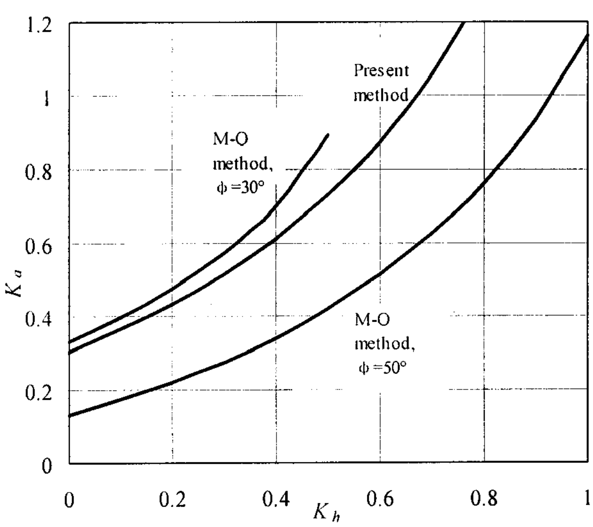
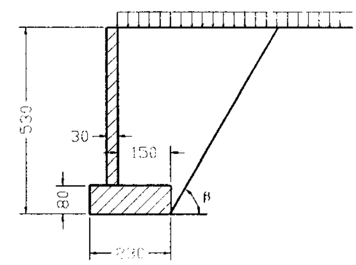


Fig. 7 The coefficients calculated by the proposed method and Mononobe-Okabe method

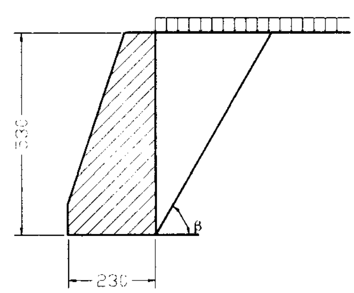
The well documented model tests on retaining walls have been carried out by Koseki et al (1998). Three types of retaining walls as shown in Fig. 8 were adopted in their experiments. The slope of active failure plane *AC* with respect to the horizontal, β, was measured for each model test. The measured β value is denoted as β_{tested} for convenience of description. Table 1 lists the data of β_{tested} measured by Koseki et al (1998). In addition, the value of β may be determined by the following formula in the form of an implicit function:

$$\frac{\sin(\beta-\phi_p+i)}{\sin(\beta-\alpha)\cos(\beta-\theta-\delta-\phi_p)} = \frac{\cos^2(\phi_p-\theta-i)}{\cos(\theta+\delta+i)\cos(\alpha-\theta)\left[1+\sqrt{\frac{\sin(\delta+\phi_p)\sin(\phi_p-\alpha-i)}{\cos(\theta+\delta+i)\cos(\alpha-\theta)}}\right]^2} \quad (7)$$

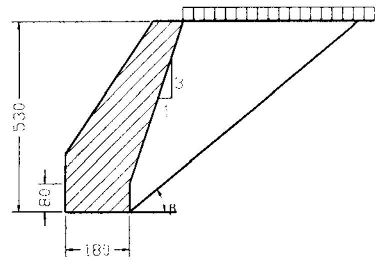
This equation was derived in the same way as Eqs. (2)–(5). The value of β thus may be determined by Eq. (7) and denoted as $\beta_{calculated}$. The results are listed in Table 1. A comparison between β_{tested} and $\beta_{calculated}$ is also made in Fig. 9. It can be seen from Table 1 and Fig. 9 that good agreements exist between the two, showing the essential effectiveness of the present method.



(a) Beam type retaining wall



(b) Gravity type retaining wall



(c) Back-inclined type retaining wall

Fig.8 Three types of retaining walls adopted in model tests *conducted by Koseki et al (1998)*

Table 1 A comparison of the tested and calculated slopes of active failure plane with respect to the horizontal

No.	Type of retaining wall	$\beta_{calculated}$	β_{tested}
S-2	beam type	53	55
S-3	gravity type	57	59
S-4	Back-inclined	53	51
S-5	Back-inclined	53	50
S-6	incline type	52	49

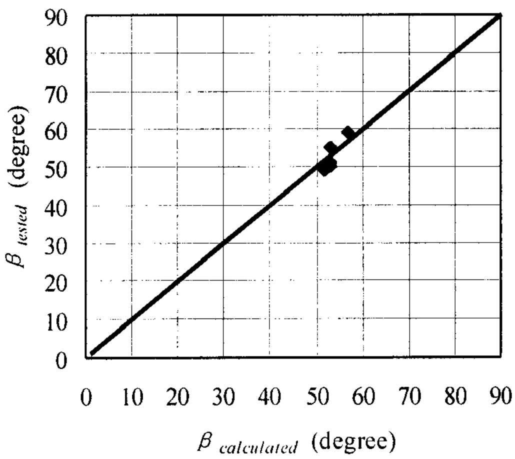


Fig. 9 A comparison of calculated and tested angles of active failure plane with respect to the horizontal

CONCLUSIONS

(1) A new pseudo-static and limit-equilibrium method was proposed for evaluating seismic (static plus dynamic) active earth pressures induced by backfill soils behind movable rigid retaining walls.

(2) The present method has the advantage over the Mononobe-Okabe method since it can take into account the effects of strain localization and post-peak reduction in the shear resistance that occur in the denser backfill soil during a strong earthquake.

(3) The present method is also better than the existing graphic method, because it allows solutions to be obtained in a very simple, logical and mechanical procedure.

(4) The earth pressure evaluation using the proposed method was checked preliminarily with previous model test results, which shows the essential effectiveness of the method.

REFERENCES

[1] Bolton, M. D. and Steedman, R. S. (1985): Modelling the seismic resistance of retaining structures, Proceedings. 11th International Conference in Soil Mechanics and Foundation Engineering, Vol. 4, pp. 1845-1848.

[2] Mononobe, H. (1924): Consideration into earthquake vibrations and vibration theory (in Japanese), Journal of Japan society of civil Engineering, Vol. 10, No. 5, pp. 1063-1094.

[3] Koseki, J., Y. Munaf, F. Matsuoka, M. Tateyama, K. Kojima and T. Sato (1997): Model tests on seismic behavior of soil retaining walls (Part 2) (in Japanese), Vol. 49, No. 11, pp. 43-46.

[4] Koseki, J., F. Tatsuoka, Y. Munaf, M. Tateyama and K. Kojima (1998): A modified procedure to evaluate active earth pressure at high seismic loads, Special Issue of Soils and Foundations, No. 2, pp. 209-216.

[5] Okabe, S. (1924): General theory on earth pressure and seismic stability of retaining wall and dam, Journal of the Japan Society of Civil Engineering, Vol. 10, No. 6, pp. 1277-323.

[6] Zhang, J.-M., Y. Shamoto and K. Tokimatsu (1998a): Evaluation of earth pressures under any lateral deformation, Soils and Foundations, Vol. 38, No. 1, pp. 15-33.

[7] Zhang, J.-M., Y. Shamoto and K. Tokimatsu (1998b): Seismic earth pressure theory for retaining walls under any lateral displacement, Soils and Foundations, Vol. 38, No. 2, pp. 143-163.